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Managing Dam Safety Risks Related to Hydraulic Structures

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ABSTRACT

The Bureau of Reclamation's Dam Safety Program manages risk for over 300 high and significant hazard dams. When risk estimates for potential failure modes indicate increasing justification for taking action to reduce risk, dam safety recommendations are typically made. The dam safety recommendations can focus on collecting additional data and performing studies to better quantify risk, or on initiating corrective actions to address well-defined risks. Dam safety recommendations within Reclamation's inventory address a number of different issues, such as those related to internal erosion, flood overtopping of dams, and seismic stability of dams. Dam safety recommendations related to hydraulic structure potential failure modes also represent a significant portion of the recommendations within Reclamation's inventory. This paper will focus on the methodology used to estimate risks for some of the more common hydraulic structure potential failure modes and will summarize the relative contribution of the risk posed by hydraulic structures to the overall risk within Reclamation's inventory of dams. Potential failure modes related to hydraulic structures include overtopping of chute walls, stagnation pressure failure of spillway chute slabs, cavitation damage of concrete flow surfaces leading to loss of concrete lining, structural failure of spillway gates, and erosion of the foundation and scour and headcutting in the downstream channel or dam/spillway foundations. The paper also includes examples of dam safety modifications related to hydraulic structure potential failure modes.

Keywords: *Hydraulic Structures, Risk, Dam Safety, Modification.*

1. INTRODUCTION

Ensuring that dams are safe and do not pose an unacceptable risk to the public are essential goals for a dam owner. Dam safety should be a key consideration during the design of a new dam and is reflected in both the design loads selected for the dam and redundant features included in the design. Once a dam is in operation, dam safety continues to be a key consideration and can be impacted by changes in the physical condition of the dam or waterways and changes in the potential loads at the dam. The Bureau of Reclamation has a robust dam safety program that utilizes risk analysis and risk assessment as foundational pieces for evaluating the safety of its dams.

Dams are evaluated thoroughly on an eight-year cycle as part of a Comprehensive Review (CR). Some of the elements of a CR include an inspection of the dam; a review of the static, hydrologic, and seismic hazard at the dam; a re-evaluation of the critical potential failure modes (PFM) at the dam; and an estimation of risk for the critical PFMs. During CRs, dam safety recommendations are often made if risks for a PFM are above the threshold values for increasing justification to take action to reduce risk (Reclamation 2011). Dam safety recommendations can also be made at the conclusion of higher level studies. Dam safety recommendations can initiate higher level analysis or studies or, if they are generated from a higher-level study, can initiate corrective action studies to evaluate modification alternatives that will reduce risk.

In 2015, there were 185 outstanding dam safety recommendations within Reclamation's dam safety program. Of these, 31 percent were related to the hydraulic structures in some way (e.g. flood overtopping of the dam or issues with the hydraulic structures). Seventeen percent of the outstanding dam safety recommendations in 2015 were specifically tied to the hydraulic structures (related to hydraulic issues associated with the spillway or structural capacity issues with the spillway during earthquakes). Dam safety risks related to the hydraulic structures make up a significant percentage of the efforts within the dam safety program. The most common issues have been related to overtopping of chute walls, stagnation pressure failure of spillway chute slabs, cavitation damage of concrete flow surfaces, and structural failure of spillway gates. Each of these common potential failure modes is discussed in the following sections. If the risk for any PFM is high enough, and if the confidence in the risk estimates is good, corrective actions to reduce the risk related to the PFM is typically pursued by Reclamation. All of the PFMs addressed in this paper have resulted in modifications to several Reclamation spillways. The type of modification and the details will vary with each dam. Examples of modifications are provided in the following sections.

2. CHUTE WALL OVERTOPPING

Spillway structures often rely on a concrete chute to safely convey spillway releases from the crest structure to an energy dissipation structure near the river channel. The spillway chute forms a rectangular or sometimes trapezoidal open channel. Spillway chute walls were typically sized for the flow depths that would occur during the design spillway discharge, plus some freeboard to accommodate variations in flow depths due to air bulking (air entrainment) and cross-waves. If the spillway chute is subjected to discharges larger than the design discharge, or if air bulking or cross waves were not incorporated properly into the design, flow depths in the chute will increase and the walls may overtop. Overtopping flows will likely initiate erosion in the wall backfill, which has the potential to progress to the point of undermining the spillway chute slab and failing the invert of the spillway. Once this occurs, headcutting can initiate and progress upstream, possibly leading to a breach of the reservoir.

2.1. Spillway Design Discharge

The discharge that the spillway was designed for will determine the flow capacity of the spillway chute and stilling basin. If current flood loadings indicate that the spillway design discharge will be exceeded for some flood events, then the flow depths in the spillway chute and stilling basin will increase and wall overtopping becomes more likely for those floods. Whether the walls actually overtop during a given flood will be influenced by both the freeboard provided in the original design and factors that may not have been accounted for in the original design, including air bulking, cross waves, or variations in boundary roughness. If the current flood loadings indicate that the spillway design capacity will not be exceeded, and if a review of the design documentation indicates that the design methods were adequate, overtopping of the chute walls will generally not be a concern.

2.2. Spillway Discharges (Depths and Durations)

Water surface profiles in the spillway can be estimated for discharges that are obtained from the routings of frequency floods. A range of discharges that correspond to given frequency floods should be evaluated to provide flow depths and velocities at selected stations in the chute and can be completed with either models or boundary layer theory calculations. Flood routings will provide information on the duration of certain discharge levels. If durations of spillway flows are limited, failure of a spillway chute that has overtopped may initiate but may not have time to fully develop into a breach of the reservoir.

2.3. Other Factors That Can Affect Flow Depths

There are a number of additional factors that can affect flow depths in a spillway chute, and these should be considered when evaluating the potential for chute walls to overtop. These factors include converging and diverging chute walls, air bulking of the flow, and cross waves in a spillway chute. The best hydraulic performance of a spillway chute is obtained when the confining sidewalls are parallel to the flow direction and the distribution of flow across the channel is relatively uniform. In order to optimize a spillway design, however, it may have been desirable to make the chute narrower or wider than either the crest structure or the terminal structure. Sidewall convergence must be made gradual to avoid cross waves, wave run-up on the walls, and uneven distribution of flow within the chute. In a similar manner, the divergence of spillway chute walls should be limited, or the flow will not spread to uniformly fill the chute. Guidance on acceptable angular variation of the flow boundaries is provided in Reclamation (1987).

Air bulking occurs where the turbulent water boundary reaches the water surface and air is introduced into the flow (entrained air) as a result of this turbulence. Under certain conditions, air bulking can significantly increase the flow depths in a spillway chute. Procedures that can be used to account for air bulking are provided in Reclamation and USACE (2015).

Cross waves can form in a spillway chute from a variety of sources: non-symmetrical entrance conditions into the spillway control structure, chute walls that converge too rapidly, piers that are introduced into the flow and then terminate, or curved chute walls. Cross waves will be superimposed on the flow depths that would occur under normal conditions and could lead to wall overtopping. For trapezoidal channels, cross waves can lead to run-up and wall overtopping sooner than for rectangular sections.

2.4. El Guapo Dam Spillway

El Guapo Dam is located on the Rio Guapo, 5 km south of the city of El Guapo, in the state of Miranda, Venezuela. The reservoir volume is 40 million m^3. The dam was constructed from 1975 to 1980. The original spillway at El Guapo Dam consisted of an uncontrolled ogee crest, located on the left abutment of the dam, a concrete chute, and a concrete hydraulic jump stilling basin. The spillway had a width of 12 m, a length of 282 m, and a design discharge capacity of 102 m^3/s. Initial hydrologic studies were based on a basin similar to the Rio Guapo basin. During construction of the spillway, the chute walls were overtopped, which triggered a new flood study. A tunnel spillway was constructed through the dam's left abutment, 250 m from original spillway.

On December 14, 1999, the reservoir was 1 m above the normal pool and 5 m below the dam crest. The radial gate on the tunnel spillway was fully open; both spillways were operating normally. Early on the morning of December 15th, the reservoir rose quickly and was 0.8 m below the dam crest. Early the next morning, the reservoir was 20 cm below the dam crest; the spillway chute walls just below the spillway crest began to overtop, and erosion of the adjacent fill initiated. By 4:30 a.m. on December 16th, cities below the dam were evacuated. At 9:00 a.m., the dam was inspected by helicopter, and the reservoir level had subsided (0.8 m below crest); people believed that flood had crested and the crisis was over. At 4:00 p.m. on December 16th, the reservoir rose again quickly; the bridge over the spillway collapsed; erosion of spillway backfill accelerated and the reinforced concrete chute, basin, and crest structure failed; but the concrete lined approach channel remained intact and controlled flows through the spillway. At 5:00 p.m., the approach channel failed and the reservoir was breached through the spillway area. El Guapo Dam never overtopped. Overtopping of the spillway chute walls initiated erosion of backfill behind chute walls and undermining and failure of the spillway chute. Headcutting progressed upstream and lead to reservoir breach. The spillway foundation consisted of decomposed rock, which was erodible (Villar 2002).

2.5. Modifications to Reduce Chute Wall Overtopping Potential

If the potential for overtopping of the chute walls exists, the solution is usually simple. The walls can be raised to contain the desired spillway flows, and the extent of the wall raise will typically be limited to only a portion of the chute. This type of modification was implemented at Reclamation's A.R. Bowman Dam in central Oregon. Based on a hydraulic model study, there was a concern for spillway wall overtopping under the increased flows above the original design discharge. Modification to address this issue consisted of installing flashboards over a 41-m length of the spillway chute. These flashboards were fabricated from 1.2-m by 2.4-m by 1-cm thick steel plates bolted to the inside face near the top of the walls. These panels extend upward 0.6 m (normal to the sloping top of wall) above the top of the walls.

3. STAGNATION PRESSURE RELATED FAILURE

Stagnation pressure related spillway failures can occur as a result of water flowing into cracks and joints within a spillway chute during spillway releases. A portion of the velocity head from the flow can be converted to an uplift pressure under the chute slab if vertical offsets exist, and water can pass through a joint without functional waterstops. If water entering a joint or a crack reaches the foundation, failure can result from excessive pressure and/or flow into the foundation. If no drainage exists, or if the drainage is inadequate, and if the slab is insufficiently tied down, the build-up of uplift pressure under a concrete slab can cause hydraulic jacking. If drainage paths are available but are not adequately filtered, erosion of foundation material is possible, and structural collapse may occur. Figure 1 depicts the development of stagnation pressures under a spillway chute slab.

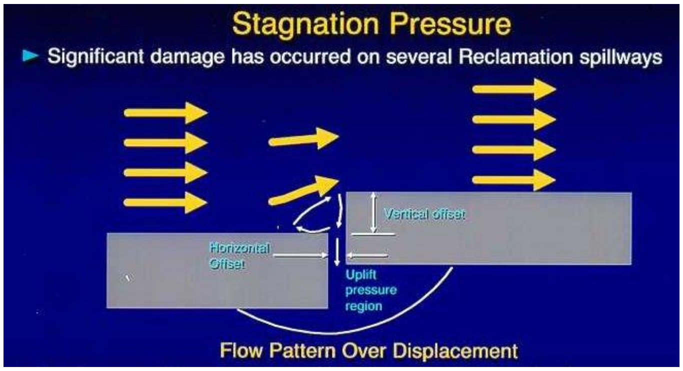

Figure 1. Development of Stagnation Pressures

Failure of a spillway chute slab, due to either hydraulic jacking or as a result of collapse after the supporting foundation is removed, can lead to scour of the spillway chute foundation, headcutting, and breach of the reservoir through the spillway crest structure.

3.1. Defensive Design Measures

Defensive design measures can prevent the failure mode from initiating or developing. Defensive design measures include the following (listed in order of decreasing effectiveness): waterstops, which can block path for water flow through joints in slabs; transverse cutoffs, which prevent vertical offsets at transverse joints and limit path for water from inside of chute to foundation; longitudinal reinforcement/dowels across chute floor joints, which minimize width of cracks and openings at joints and may prevent offsets; anchor bars, which provide resistance to uplift pressures lifting slabs off foundation; filtered underdrains, which relieve uplift pressures that can be generated under slabs (filtering prevents movement of foundation materials into the drainage system and initiation of foundation erosion); and insulation, which insulates the drainage system and prevents it from freezing and also prevents frost heave locally. An absence of or inadequate defensive design measures can allow initiation and progression of this failure mode.

3.2. Reclamation Research on Stagnation Pressure Potential

Reclamation conducted research in 2007 to evaluate different spillway chute transverse joint geometries and their effects on the stagnation pressures that could be generated beneath a spillway chute slab, as well as the flows that could be transmitted underneath the slab (Reclamation 2007). Both drained and undrained conditions were evaluated. Figure 2 is an example of the testing results, which depict uplift pressures that were generated for a transverse joint with a 1.3-cm gap, sharp edged geometry and drained conditions for a variety of offset dimensions that encroach into the flow from the channel boundary (downstream slab raised with respect to upstream slab).

3.3. Big Sandy Dam Spillway

Big Sandy Dam is located on the Big Sandy Creek, 72 km north of Rock Springs, Wyoming. The earthfill embankment dam was completed in 1952. The spillway is located on the right abutment of the dam and consists of an uncontrolled concrete side-channel crest structure and a concrete chute and stilling basin. The spillway is founded on thinly bedded to massive siltstone and sandstone. A zone in the foundation below the spillway inlet structure contains open joints and bedding planes, which allowed reservoir water to seep under the spillway chute floor. The spillway chute was designed with an underdrain system and anchor bars grouted into the foundation rock, but waterstops and continuous reinforcement were not provided across the contraction joints. Deterioration of the concrete slab occurred shortly after the dam was put into service. Cracking occurred in the chute slabs due to excessive water and ice pressures along the foundation-concrete slab interface, and some of the slabs heaved and were displaced off the foundation, creating offsets into the flow. The spillway operated from 1957 to 1982 without incident, but a chute floor slab failed in June 1983 due to uplift pressures from flows of 11 m^3/s (Hepler and Johnson 1988). The potential failure mode did not progress beyond the spillway slab jacking, primarily due to the erosion resistance of the underlying foundation relative to the energy of the spillway release flows. Calculations were performed to confirm that the failure was the result of stagnation pressures being generated under the chute slab. The calculations also showed that with fully effective anchor bars, the slab would not have failed. The uplift pressures assumed in the calculations were estimated from extrapolated laboratory tests (Hepler and Johnson 1988). From observations after the failure, it was noted that the anchor bars exposed beneath the slab were not coated with grout, indicating that the anchor bar capacity was not fully developed. This case history illustrates that both well thought out details and good construction procedures are needed to achieve performance.

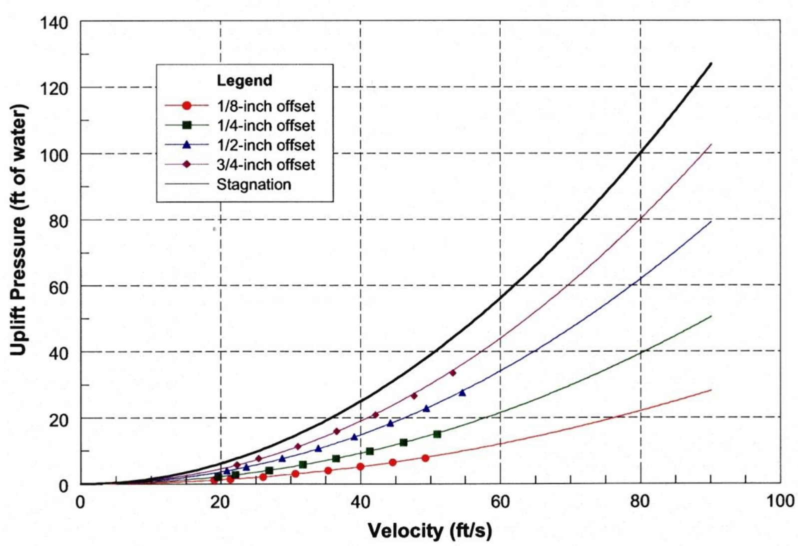

Figure 2. Mean uplift pressure, sharp-edged geometry, vented cavity, 1.3 cm (1/2-inch) gap (Reclamation 2007)

3.4. Modifications to Reduce Stagnation Pressure Failure Potential

Modifications to prevent a stagnation pressure failure along a spillway chute typically involve replacing a portion or all of the spillway chute and incorporating defensive design measures into the spillway chute. Figure 3 shows the typical design details that are beneficial in reducing the potential for stagnation pressure failure. These design details were implemented in spillway chute modifications at Reclamation’s Hyrum Dam in Utah and Big Sandy Dam in Wyoming.

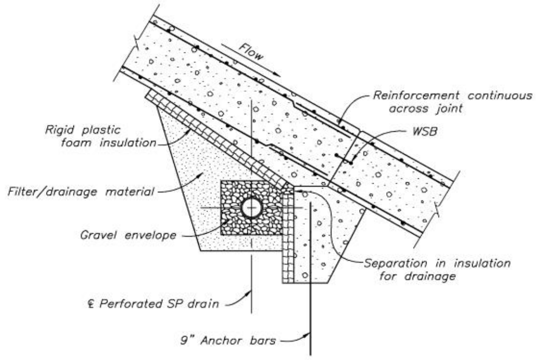

Figure 3. Defensive Design Measures for Stagnation Pressures

4. CAVITATION DAMAGE

Cavitation is the formation of vapor cavities in a liquid. Cavitation occurs in high-velocity flow, where the water pressure is reduced locally because of an irregularity in the flow surface. As the vapor cavities move into a zone of higher pressure, they collapse, sending out high-pressure shock waves (see Figure 4). If the cavities collapse near a flow boundary, there will be damage to the material at the boundary. Cracks, offsets, and surface roughness can increase the potential for cavitation damage. The extent of cavitation damage will be a function of the cavitation indices at key locations in the spillway chute or tunnel and the duration of flow. If the spillway lining is completely failed, the spillway foundation will be exposed, which can lead to scour and headcutting or expansion of the eroded area. These mechanisms could lead to breach of the reservoir. In most cases, this failure mode is unlikely to progress to the point where dam failure occurs due to the long flow durations that are required to cause major damage to concrete linings.

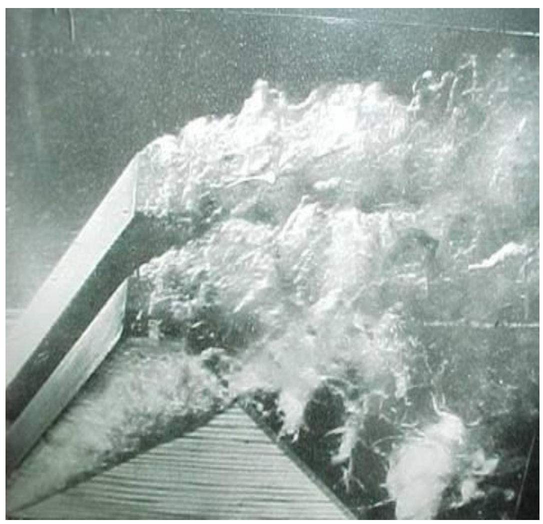

Figure 4. Cavitation Created in Low Ambient Pressure Chamber

4.1. Condition of Concrete in Spillway

Cracks, offsets, surface irregularities, and/or open joints in chute slabs (or tunnel linings) and the lower portions of chute walls exposed to flow may allow this failure mode to initiate. The geometry of the flow surface irregularities will affect the initiation of cavitation. The more abrupt the irregularity, the more prone the spillway will be to the initiation of cavitation. Concrete deterioration in the form of alkali-silica reaction, freeze thaw damage, and sulfate attack can exacerbate this PFM by creating surface irregularities and/or offsets at damaged areas.

4.2. Cavitation Indices

Cavitation indices can be used to evaluate the potential for cavitation damage in a spillway chute or tunnel. The cavitation index is defined as follows:

$$\sigma = \frac{P - P_v}{\frac{\rho V^2}{2}} \tag{1}$$

where P = pressure at flow surface (atmospheric pressure + pressure related to flow depth), P_v = vapor pressure of water, ρ = density of water, and V = average flow velocity.

There is the potential for cavitation damage to initiate when the cavitation index, σ, is between 0.2 and 0.5 for typical concrete, but significant damage is typically associated with cavitation indices less than 0.2 and long durations of spillway operation. For large features that are introduced into the flow abruptly (such as stilling basin baffle blocks or splitter walls), cavitation damage can occur when the σ is as high as 1.0 or greater. Additional information on the potential for cavitation damage can be found in Falvey (1980).

4.3. Aeration of Flow

The introduction of air into spillway flows reduces the potential for cavitation to damage concrete surfaces. Aeration reduces the damage that occurs from collapsing vapor cavities. If the flow is not naturally aerated, measures can be taken to introduce air into the flow at critical locations along a spillway.

4.4. Glen Canyon Dam Spillway

Glen Canyon Dam is located on the Colorado River in northern Arizona. The dam, completed in 1964, is a constant radius, thick-arch concrete structure. Spillways are located at each abutment, and each consists of a gated intake structure regulated by radial gates, a concrete lined tunnel through the soft sandstone abutments, and a deflector bucket at the downstream end. Each spillway tunnel is inclined at 55 degrees, with a vertical bend and a horizontal section. The spillways experienced significant cavitation damage during operation in June and July, 1983 during flooding on the Colorado River system, when the reservoir filled completely for the first time and releases were required. The cavitation damage was initiated by offsets formed by calcite deposits on the tunnel invert at the upstream end of the elbow. Both spillways were operated at discharges up to about 850 m^3/s. Cavitation indices of the flow in the area where damage initiated in the left spillway ranged from about 0.13 to 0.14. The cavitation indices of the deposits along the tunnel (indices at which cavitation was likely to occur) ranged from 0.64 to 0.73. The worst damage occurred in the left tunnel spillway: a hole 11-m deep, 41-m long, and 15-m wide was eroded at the elbow into the soft sandstone (Burgi and Eckley 1987).

4.5. Modifications to Mitigate Potential for Cavitation Damage

An effective means of mitigating the potential for cavitation damage is to entrain air into the flow. This dramatically reduces the damage that can occur on flow surfaces. This is typically accomplished by separating the flow from the flow surface (through the use of a ramp) and then introducing air on the underside of the flow jet. An example of this is shown in Figure 5, which shows the air slot installed at Glen Canyon Dam.

Figure 5. Air Slot at Glen Canyon Dam

The 1.2- by 1.2-meter air slot at Glen Canyon Dam extends over the lower three-quarters of the tunnel circumference, which ensures that adequate air entrainment and good distribution of the entrained air occurs in the flow.

5. SPILLWAY RADIAL GATE FAILURE

Gated spillways can create a vulnerability during floods if the gates cannot be reliably operated to their full capacity. Gate reliability can be impacted by failure of power supplies, binding of gates, or failure of components of the hoist system. While gate reliability issues can increase the probability of a dam overtopping during a flood, this paper focuses on spillway gate structural issues that can lead to an uncontrolled release of the reservoir. The most common type of spillway gate in Reclamation's inventory is a radial gate. Radial gates consist of a cylindrical skinplate reinforced by vertical or horizontal support ribs, horizontal or vertical girders, and the radial arm struts that transfer the hydraulic loads to the gate trunnions. Radial gates rotate about their horizontal axis during opening/ closing operations. These gates are generally large, and the capacity through a failed radial gate can often generate large uncontrolled releases. If the ultimate capacity of the gate is exceeded, a radial gate can fail rapidly. Most of the load on a radial gate is from the reservoir, but additional loads can be generated from trunnion pin friction for a gate that is being operated or from hydrodynamic and inertial effects during an earthquake. Reclamation has documented procedures for estimating risk from radial gate failure both during normal conditions and seismic conditions (Reclamation and USACE 2015).

5.1. Interaction Ratio

A key consideration in evaluating the potential for a radial gate to fail is the interaction ratio (IR), which is used to assess the stability of a structural member subjected to axial compression and bi-axial bending. Instability of the member will occur in accordance to AISC (AISC 2011) when the loadings result in the interaction ratio exceeding 1.0. When evaluating stability for a risk analysis, all load and resistance factors are equal to 1.0 so the actual load carrying capacity of the structure is evaluated. The following equations can be used to calculate the IR for a radial gate element:

$$IR = \frac{P_u}{P_n} + \frac{8}{9}\left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}}\right) \qquad for\ \frac{P_u}{P_n} \geq 0.2 \tag{2}$$

$$IR = \frac{P_u}{2P_n} + \left(\frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}}\right) \qquad for\ \frac{P_u}{P_n} < 0.2 \tag{3}$$

where P_u – required axial strength, P_n – the available axial strength equals the nominal compressive strength, M_u – required flexural strength, M_n – the available flexural strength equals the nominal flexural strength, and subscripts x and y relating to strong and weak axis bending, respectively

5.2. Other Considerations for Radial Gate Failure

In addition to the IR, other factors should be considered in a risk analysis evaluating radial gate failure. These include the gate arrangement and structural conditions, the frequency of inspecting and exercising the radial gates, and the potential for the radial gate bushings to fail and lead to increased trunnion friction. All of these factors have the potential for either reducing the structural capacity of the gates or increasing the loading on the gates. Guidance on how to incorporate these elements into a risk analysis is provided in Reclamation and USACE (2015).

5.3. Failure of Radial Gate at Folsom Dam

Folsom Dam was designed and constructed by the U.S. Army Corps of Engineers between 1948 and 1956. The dam was transferred to the Bureau of Reclamation for operation and maintenance in 1956. The dam consists of a concrete gravity section across the river channel, flanked by long earth fill wing dams. The concrete dam has a gated overflow spillway section that is regulated by eight radial gates: five service gates and three emergency gates. One of eight large spillway radial gates failed at Folsom Dam in California during reservoir releases on July 17, 1995. The gate failure occurred with a nearly full reservoir releasing a peak flow of about 1130 m^3/s. No injuries or fatalities occurred as a result of the gate failure.

Gate No. 3 was being operated at approximately 8 a.m. on July 17, 1995, to maintain flow in the river during a powerplant shutdown. As the gate was opened, it was allowed to stop at 15 cm automatically and again at 30 cm. The auto-stop function was overridden (normal procedure) with no stop being made at the 60-cm level. As the gate opening approached 73 cm, the gate operator felt an “unusual vibration” and he stopped the gate hoist motor. As the operator turned to check the gate, he saw the right side of the gate swing open slowly, like a door hinged on the left side, and saw water pouring around both sides of the gate leaf (see Figure 6). The time from the operator’s initial awareness of the vibration to observing gate displacement and uncontrolled flow of water was estimated to be no more than 5 seconds.

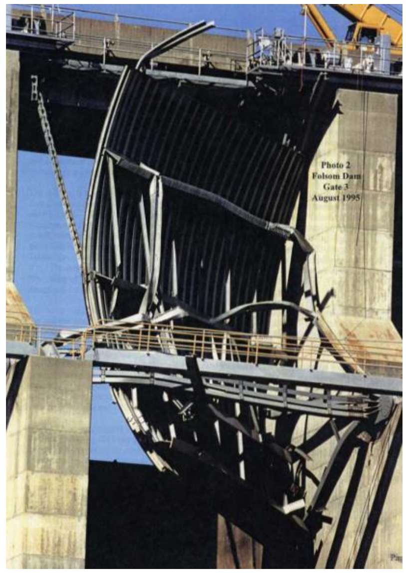

Figure 6 – Failure of Radial Gate No.3 at Folsom Dam

Following the failure of Gate No.3, a multi-disciplinary, multi-agency forensic team was formed to investigate and determine the cause of the failure. The team identified the two main causes of the gate failure as insufficient stiffness and strength in critical structural gate arm members and increased trunnion friction by corrosion of the steel trunnion pins.

5.4. Modifications to Reduce Structural Failure of Spillway Radial Gates Potential

There are several options for reducing the potential of spillway radial gate failure. If the issue is driven by trunnion pin friction, the bushings can be replaced with bushings that reduce the trunnion pin friction. If the structural capacity of key gate elements is lacking, gate elements can be modified by increasing the steel section. At Reclamation's Bradbury Dam in California, the flanges of the WF beams that form the radial gate arms were thickened by welding steel plates onto the existing flanges. This increased the section of the beams and increased their structural capacities.

6. CONCLUSIONS

Risk analysis and risk assessment are key elements of Reclamation's Dam Safety Program. This approach has allowed Reclamation to identify the most critical issues within its inventory and to focus efforts to reduce overall dam safety risks. This process has identified a number of deficiencies associated with the hydraulic structures at Reclamation dams. Modifications have been implemented to address these issues and reduce the chance of a hydraulic structure contributing to dam failure.

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